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Christchurch Women's Hospital: Performance Analysis of the Base-Isolation System During the Series of Canterbury Earthquakes 2011-2012

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Abstract: Live monitoring data and simple dynamic reduced-order models of the Christchurch Women's Hospital (CWH) help explain the performance of the base isolation (BI) system of the hospital during the series of Canterbury earthquakes in 2011-2012. A Park-Wen-Ang hysteresis model is employed to simulate the performance of the BI system and results are compared to measured data recorded above the isolation layer and on the 6th story. Simplified single, two and three degree of freedom models (SDOF, 2DOF and 3DOF) show that the CWH structure did not behave as an isolated but as a fixed-base structure. Comparisons of accelerations and deflections between simulated and monitored data show a good match for isolation stiffness values of approximately two times of the value documented in the design specification and test protocol. Furthermore, an analysis of purely measured data revealed very little to no relative motion across the isolators for large events of moment magnitude scale (M_w) 5.8 and 6.0 that occurred within 3 hours of each other on December 23, 2011. One of the major findings is that the BI system during the seismic events on December 23, 2011 did not yield and that the superstructure performed as a fixed-base building, indicating a need to reevaluate the analysis, design and implementation of these structures.

Keywords: base isolators, base-isolation, Canterbury earthquakes, shear-wall model, Park-Wen-Ang model, real-life data, structural monitoring.

Introduction

Christchurch, New Zealand was struck by a series of major earthquakes and aftershocks beginning September 2010 [Nicholls 2012], including the moment magnitude scale 6.3

(M_w6.3) event February 22nd, 2011 that caused major damage and resulted in the loss of 185 lives [ECRC 2011, EERI 2011]. The Christchurch Women's Hospital (CWH) is the sole base isolated structure in New Zealand's South Island, and suffered damage beyond expectations [McIntosh 2012, stuff 2012]. Observed damage was attributed to the Darfield (4 September) and Christchurch (22 February) events. Subsequent events in June and December 2011 were not assumed to have contributed significantly more to the damage [stuff 2012]. A directly related damage of the structure to the measurements of the 23 December event(s) cannot be provided as the building was still in service and no formal observation report was filed by the building engineers. Understanding the seismic performance of the base-isolated CWH structure will provide greater knoweldge of the seismic risk in Christchurch and inform the extensive rebuild of the city.

This project and its analysis of real-life, large ground motion induced data is unique, as far as the authors know. Extensive literature exists on theoretical and experimental investigations of isolators, retrofited buildings [Guo 2014, Bailey 1988] and base-isolated structures on shaking tables (usually scaled or full-scale, but empty) [Lakshmanan 2008, Madden 2002]. However, to the best of the authors knowledge, the performance of a fully operational building subject to real-life seismic events has not been investigated and documented before, which is likely a result of there being relatively few base isolated structures, far fewer subject to major ground motions, and thus likely none of those being monitored. An older list, compiled by Buckle et al, exists about base-isolated structures in the United States of America [Buckle 2000] and worldwide [Buckle 1990] to which the CWH performance could be compared to in the future, if some were monitored and then subject to a large seismic event.

The closest project to which the CWH performance could be compared to is probably the 5-story experimental building project led by Hutchinson et al. [UCSD2 2012, Pantoli 2015, Chen 2013, Chen 2015]. It is described as including a fully functional elevator, prefabricated metal stairs, partition walls, ceilings, synthetic stucco and precast concrete cladding exterior façades, as well as mechanical, electrical and plumbing systems and medical equipment. It also included two medical floors, a computer server room, living space and utilities levels, and was thus, in sum, a very realistic building. However, it lacked building services across the isolators, adjacent buildings as well as mold covers, relative to the structure investigated in this work. In addition, the isolators protected the structural and non-structural elements, in contrast to the results here, so the lack of isolation cannot be compared.

A network of instruments installed at the CWH in September 2011 recorded acceleration and displacement data during subsequent seismic activity [Gavin 2012, Gavin 2013]. A study of these records [Sridhar 2012, Sridhar 2013] indicated that the base isolated structural response was that of a linear, fixed-base structure. This analysis identifies a series of structural models using real-life data to assess the validity of this initial result compared to the expected, isolated design response.

Structural models range from highly non-linear finite element models [Crisfield 1991, Kiyohiro 2000, Oztorun 1998] to simple, linear single degree of freedom models [Calio 2003]. Previously, similar analyses were used to assess the failure of the Canterbury Television Building [CEL 2012]. This analysis identifies models, from data, that would admit both the fixed-base conditions observed or the base-isolated system designed. Hence, identification of the model over a limited number of larger (M_w6.0) events and one or more lower in magnitude events (M_w4.0-4.5) will quantify the dynamic system response over a range of seismic inputs. Differences from its anticipated as-designed behaviour will provide the quantified input to reconsider how such systems are designed for this region in future.

Methods and Models

Christchurch Womens Hospital Structure

CWH is a 10-story, 75 m x 32 m structure (Figure 2) opened in March 2005 [HCG 2005]. The CWH building has been designed for an alpine fault rupture of magnitude >M_w8.0 according to Uniform Building Code 1997 [UBC 1997]. The base isolation system was designed adopting a return period of 2000 yrs (R=1.7 (CATII, 50 yrs Design Life)) for the maximum credible event and 500 yrs for the maximum probable earthquakes or design basis earthquakes. Figure 1 depicts the design spectra for the structure.

The base-isolator system has been designed to shift the natural, fundamental period of the structure to 3 s [Gavin 2010, HCG 2001]. The fundamental period of the equivalent fixed-base structure is 1.2 sec. The superstructure is supported on a concrete raft foundation slab, supported by 41 lead rubber bearings and 4 sliding pot bearings. Tables 1 and 2 list the design specifications of a single lead rubber and sliding pot bearing, respectively, used in the CWH [HCG 2001]. All lead rubber and sliding bearings have identical parameters and have been tested before installation [Oiles 2002]. A perimeter concrete frame resists seismic loads aided by lateral load resisting V-bracing over the first lower 4 storeys (Figure 2). The CWH building is linked to the adjacent Christchurch Hospital in the basement and at the four above-ground floors via corridors with a sliding gap and sliding cover plates to provide seismic separation. The detailed design was obtained from structural drawings and the designers report written for the owner and council during the consenting process.

Acceleration and Displacement Measurements

Sensors comprised 4 tri-axial and 3 uni-axial accelerometers, as well as 3 displacement sensors, installed September 2011 [Gavin 2012], where specific sensor locations are detailed in Table 3 and Figure 2. Sensors were located across the isolation layer (above/below BI) and the 6th story. It is thus possible to integrate and correct acceleration data across the isolation layer to get relative displacement (and velocity) of the slab and foundation, as well as to measure this displacement directly. Sensors in two corners ensure that both translatory and rotational (torsional) responses of the superstructure can be measured.

Recorded Ground Motions Used

Over 100 seismic events were recorded since the installation of sensors in 2011. This analysis is limited to the M_w5.8 and M_w6.0 earthquakes recorded on December 23rd, 2011 [Gavin 2013], and one arbitrarily selected M_w4.0 event [GNS 2012] also obtained on December 23rd, 2011. Accelerations in the NE and SW corners were effectively identical in both directions [Sridhar 2012], indicating a single direction response and the absence of torsion.

Models

A Park-Wen-Ang (PWA) nonlinear, hysteretic model [Park 1986, Wen 1976] is used to model the BI system based on prior work by Gavin et al. [Gavin 2010], and uses a simplified superstructure model in 2 dimensions (x, y). Presented is the behaviour of the structure in the y direction, decoupled from the perpendicular x direction (see Figure 2).

This consideration is justified by the significantly stiffer ($\times 2.5$) nature of the building in x direction (due to the aspect ratio) than in its y direction. The mass normalized shear force of the isolation system f_y is thus [Gavin 2010]:

$$f_y = C_y g\left((1-\kappa)z_y + \kappa\frac{y_b}{D_y}\right), \tag{1}$$

where the design parameters C_y, D_y, and κ are defined in Table 4 and g is the gravity constant. y_b in (1) is the variable describing the motion of the building in y-direction. The hysteretic variable z_y is defined [Gavin 2010]:

$$\dot{z}_y = \frac{\dot{y}_b}{D_y} - (\alpha\,\mathrm{sign}(z_y\dot{y}_b) + \beta)\frac{z_y^2\dot{y}_b}{D_y}\ , \tag{2}$$

where $\alpha = 0.8$ and $\beta = 0.2$ [Gavin 2010] are dimensionless values that determine the shape of the hysteresis.

The governing equation of motion for the combined superstructure and isolation system is then

$$\ddot{y}_b + 2\zeta\dot{y}_b\sqrt{\frac{C_y g}{D_y}} + f_y = -\ddot{y}_g. \tag{3}$$

Equation (3) describes a motion of a rigid block (single degree of freedom (SDOF) system) on a BI system, fully ignoring the flexible nature of the superstructure; ζ is the damping ratio.

The SDOF system assumes that the superstructure mass $m_{\mathrm{total}} = 2.7\cdot 10^7$ kg is perfectly rigid and sits on top of the BI system with effective stiffness $k_b = 7.3789\cdot 10^8$ N/m.

Equation (3) thus captures the design approach with a flexible isolation layer and rigid superstructure.

A two degree of freedom (2DOF) model considers two flexible parts, namely the isolation layer and the superstructure between top and bottom floor. The superstructure is considered a linear shear structure and the isolator as modeled in Eqs. (1)+(2). Figure 3a shows a schematic diagram of the 2DOF model.

Finally, a three degree of freedom (3DOF) model (see Figure 3b) is evolved from the 2DOF model and considers a linear superstructure of 2 degrees of freedom and one DOF representing the BI system. The 2DOF superstructure model reflects the two distinctively different natures of effective stiffnesses in the structure. The first four lower floors have a V-brace support while the upper three floors are without the V-brace supports, which justifies the assumption that the lower part of the building is stiffer than the upper part (in the considered direction).

Parameter Identification

The isolator properties are taken from previous work by Gavin et al. [Gavin 2010]. The total mass is divided with approximately 10% assigned to a degree of freedom just above the isolation layer and the remaining 90% to the one or two DOFs assigned to the superstructure. Thus, for the 2DOF and 3DOF systems the total mass is split by estimating the ratios to $m_{\mathrm{aboveBI}} = 0.16\, m_{\mathrm{tot}}$ and $m_{\mathrm{aboveBI}} = 0.1\, m_{\mathrm{tot}}$, $m_{\mathrm{l}} = 0.6\, m_{\mathrm{tot}}$, $m_{\mathrm{u}} = 0.3\, m_{\mathrm{tot}}$, respectively, according to the first natural frequencies of the real structure. The structural damping for the superstructure uses a Caughey damping matrix [Paultre 2010] with 5-15% damping (for further details see Table 5). Numerical simulations of the multi-degree of freedom systems is performed using a Runge-Kutta method.

Stiffness values for the superstructure are identified from the SDOF model given an isolation layer stiffness (k_b). The process assumes no additional stiffness due to the non-structural walls and panels and that column stiffnesses are identical for a given DOF to find the effective structural stiffness (k_s). Thus, estimating Young's modulus of concrete as approximately 26 GPa and multiplying the stiffness (k_s) by the number of 42 columns of the CWH, the inter-story stiffness is approximated to be of the order of magnitude 10^9 N/mm for the 2DOF case, which is modified in the 3DOF model. Thus, identification involves a simple gradient descent method based on repeated re-simulation to minimise sum squared error between measured and simulated responses by modulating the value(s) of k_s for the superstructure DOFs.

Analyses

The purpose of the SDOF analysis is to match standard assumptions that an isolated structure has little or no flexible superstructure motion. The M_w4.0 event provides the linear properties of the isolator. The M_w5.8 and M_w6.0 events are used in SDOF analysis to confirm the linearity and show if the assumptions regarding the superstructure motion held at larger events, which would indicate satisfactory isolation performance.

The 2DOF and 3DOF cases are identified to account for any superstructure motions that the SDOF case misses. These cases will more precisely quantify how well (or poorly) that assumption is held if the resulting stiffnesses are compared to estimated or calculated stiffnesses from the design. A far better match to all measured displacements and accelerations is achieved.

Results and Discussion

SDOF Model Analysis

The stiffness and damping parameters of the SDOF model from Table 2 were modified to best match the measured response for the M_w5.8 event as the original parameters were not able to match the data, as seen in Figure 4. While the results in Figure 4a clearly show a softer, lesser damped behaviour with original design data (with 50 mm displacement across the isolators), the recorded data in Figure 4b only show peak displacements of 14.3 mm and a different damping mechanism. A simulation with modified parameters for the BI system show a linear behaviour with an equivalent stiffness of $k_b = 2.96 \cdot 10^9$ N/m compared to the original designed specification value of $k_b = 7.38 \cdot 10^8$ N/m, which is an increase of approximately 300 % or 4 times stiffer isolator.

Hence, for a relatively large magnitude event, the isolation layer is effectively far stiffer than predicted by the design specification. In addition, the response is effectively linear as very little damping is observed, and results are similar for the M_w6.0 event (not shown). The M_w4.0 event (Figure 5) shows far lesser displacements, as expected, and the motion across the isolation layer clearly obeys linear dynamic laws. Thus, the isolation layer can be treated to behave linearly with equivalent stiffness and damping quantities.

Figure 6 shows the measured and SDOF modeled accelerations just above the base isolators (b), d)) and at the 6th story (a), c)). The left two panels a), b) use the original design isolator stiffness, $k_b = 7.38 \cdot 10^8$ N/m, and structural mass, $m = 2.7 \cdot 10^7$ kg, and the two right panels c), d) use the modified stiffness value $k_b = 2.96 \cdot 10^9$ N/m used to match the peak displacement in Figure 4b. The lower, original stiffness matches accelerations above the isolator better than the higher stiffness (panels b), d)). However, neither value yields a good and acceptable match. The 6th story results are somewhat better, qualitatively, but the differences are large enough in both cases to indicate further flexible behaviour within the superstructure, between the floors just above the isolators and the 6th story, and thus justifying further investigations using 2DOF and 3DOF models.

2DOF Model Analysis

Figure 7 shows the simulated accelerations of the 2DOF model and corresponding measured data at the 6th story (a) and above the base isolator (b). The quality of the match is better compared to that of the SDOF model. The model uses the modified isolation layer stiffness as identified with the SDOF model (Figure 7 c, d), and identifies a value for the structural stiffness, $k_s = 8.63 \cdot 10^8$ N/m, which is $1.17\times$ the originally specified isolator stiffness, but still less than the identified isolator stiffness, although they are of similar magnitude.

Hence, the 2DOF model captures a flexibility in the superstructure observed in the data, with similar results (not shown) for the M_w6.0 and M_w4.0 events. The 2DOF model analysis shows that the equivalent stiffness values of isolation layer and structure were effectively similar, rather than one being much lower than the other as might be expected by design. However, there are still some discrepancies that might be further mitigated by the 3DOF model, particularly towards the end of the record which represents the free vibration of the BI system and superstructure.

3DOF Model Analysis

Figure 8 shows the results of the 3DOF analysis for both the M_w5.8 event shown for the prior cases, as well as for the M_w6.0 event to illustrate the robustness of our findings. The results show very good agreement throughout the record (initial as well as free responses) for both events and for both, the level above the base isolator and at the 6th story. Table 6 shows the peak accelerations for each event and the absolute error. Thus, the 3DOF model - which includes the distinction of effective stiffnesses between lower and upper floors with and without V-braces (Figure 2), respectively - adds further dynamics that explains the observed behavior quantitatively.

The identified values for the lower and upper story stiffnesses in the 3DOF case are $k_l = 8.12 \cdot 10^8$ N/m, which is $1.1 \times$ the original base isolator stiffness specification and lower than the identified value, and $k_u = 2.95 \cdot 10^8$ N/m, which is lower than either isolator stiffness value specified or identified from the SDOF case. Both these results are again contradictory to what might be expected for a base isolated structure, and thus show that the as-implemented isolation system was much stiffer in use than as originally designed.

A final comparison for the M_w5.8 event is shown in Figure 9, which shows the Fourier spectra of the acceleration response for the 6th story and the floor above the base isolator. The results of measured data and identified model are very similar up to 1.5 Hz. Above 1.5 Hz the spectra for the measurement just above the isolation layer are still in good agreement. However, the 6th story spectra drops to zero for the model and the match is no longer good above 1.5 Hz. This result is expected as the 3DOF model has natural vibration modes at 0.72, 1.36 and 3.66 Hz, and thus does not have the ability to offer the same frequency content as measured. A better match for a broader bandwith would require further degrees of freedom.

Summary and Limitations

The overall results show that the isolation layer as designed and as implemented behaved very differently. The as designed specifications showed a much lower stiffness and a lower yield point than was observed in two relatively large events on December 23, 2011.

As a result, the isolation layer had almost no relative displacement over the records and did not isolate the system from the motions.

These outcomes are evident in the model-based analyses presented. The SDOF model identifies the higher stiffness of the isolators directly and the higher yield force implicitly via a lack of observed yielding displacement in the measurements. As validation, Figure 10 shows the input accelerations and resulting relative displacements across the isolator for both large events on December 23, 2011.

The relative motion across the BI system is very small (see e.g. Figures 10d, 11b), showing lack of isolation of the superstructure from the ground motion. The strong motion input portions show only a few displacements about 3-5 mm, which is very small compared to the isolator size and represent less than 1-2% strain accross the isolator. The smaller second shock for the M_w5.8 event (right panels in Figure 10) indicates that smaller events would have no effective isolation layer motion (as is also confirmed in Figure 5 for the M_w4.0 event).

The 2DOF and 3DOF model analyses clearly show that there was a flexible, essentially fixed based response of the superstructure that was not expected by design. The 3DOF model clearly captures all relevant dynamics that were measured, while the 2DOF approach offers a good fit to data. In both cases, the identified superstructure stiffness values were of the same order of magnitude and lower than the identified isolator stiffness value. Hence, no isolation would be expected even for magnitude >M_w6.0 events.

The simple models used in this analysis do not capture all observed dynamics, of course. A more comprehensive model would capture more of the data, as noted in Figure 9. However, the fundamental dynamics are captured and the peak accelerations predicted by the 3DOF model in Table 6 are within 7-11%, which is good for such a simple model. More detailed modeling would likely reveal greater insight and resolution compared to the stiffness and damping values used here, but would not change the fundamental conclusions.

Conclusions

The main results of this study include:

- The CWH structure performed as a fixed base structure rather than an isolated structure during large magnitude seismic events, despite having been designed with methods proven elsewhere. This outcome indicates a greater need for study around the design and implementation of base isolated, critical infrastructure to ensure they perform to expectations.
- A parameter set for a 3DOF model was identified, which captures observed dominating dynamics. The analysis explains the damage observed in this structure that resulted in part from the increased upper story accelerations observed as a result of there being no reduction of input acceleration across the base isolation layer. The approach used here is simple and general.

The underlying reasons for the observed performance of the base isolators in the CWH remain speculative. However, based on the analysis, the authors rank multiple reasons from the most to the least probable as follows:

1. The CWH building has been designed for an alpine fault rupture of magnitude >M_w8.0. The base isolation system was designed adopting a return period of 2000 yrs (R=1.7 (CATII, 50 yrs Design Life)) for the maximum credible event and 500 yrs for the maximum probable earthquakes or design basis earthquakes. However, the PGA of

~0.2g should have led to isolating behaviour as the design curve for the structure intended yield of the lead rubber bearing core at 0.03g (3% building weight), [Gavin 2010]. Unlike other base-isolated structures, the system of base-isolators in the CWH is not a combination of rubber bearings with and without lead core, but all 41 bearings are identical, consisting of rubber and lead, therefor concluding that the shear force is perhaps too high by design, given the intended design performance.

2. The observed performance could be due to poor/variable bearing design or construction. An unproven indication for this is the rather large value of the design shear force at total design displacement of 740 kN for a single LRB (see Table 1).
3. While the CWH building was designed as a stand-alone structure, it was built in to a complex of existing fixed-based buildings. It was directly connected to the adjacent Christchurch General Hospital (CGH), with moat covers, service ducts and a three-story air bridge, which was added later on. These additional structural parts, while isolated by flexible gaps, could have contributed to changed lower story motion behaviour in larger events, particularly as there was observed damage across these gaps [McIntosh 2012].

Point 1. in particular could have caused a change in apparent stiffness characteristics. In addition, it is possible that the base-isolator system as designed behaved differently on top of soil with varying properties (soft and stiff, or primarily soft patches). Long period accelerations and liquefaction debris observed in the isolation galley suggest that soft soils may have contributed to this behavior [EERI 2012]. Inter-building connections between CWH and CGH increased the overall stiffness of the BI-system. The authors currently do not know how interactions with locally soft/weak soils and with adjacent/coupled structures affect the performance of seismically isolated structures, and how do these interactions scale with shaking intensity.

Hence, main results indicate a need to reconsider how base isolation is designed for structures of this type, at least in Christchurch.

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Table 1: Design parameters of one lead rubber bearing

parameters	LRB1
total design displacement [mm]	265
total maximum displacement [mm]	420
compression stiffness [kN/mm]	1,794
design shear force at total design displacement [kN]	740
design area of hysteresis loop at total design displacement [kNmm]	366,600
average (DL+SLL) [kN]	3,495
average (DL+SLL+E_{DBE}) [kN]	4,466
average (DL-E_{DBE}) [kN]	2,166
maximum (DL+LL) [kN]	4,417
maximum (DL+SLL+E_{MCE}) [kN]	6,570
minimum (DL-E_{MCE}) [kN]	357

Notes: 1) DL - dead load; 2) SLL - serviceability live load; 3) LL - live load; 4) E_{DBE} - total design displacement (design basis earthquake); 5) E_{MCE} - total maximum displacement (maximum considered earthquake); 6) design shear force is calculated as $F = Q_d + K_r\Delta$ where Q_d is the isolator characteristic strength, K_r is the stiffness and Δ is the DBE displacement; 7) area of hysteresis loop is calculated as $4Q_d(\Delta - \Delta_y)$ where Δ_y is the yield displacement of the isolator

Table 2: Design parameters of one sliding bearing as used in the CWH

parameters	pot1
average (DL+SLL) [kN]	4,986
maximum (DL+LL) [kN]	5,768
maximum (DL+SLL+E_{MCE}) [kN]	11,270
maximum motion [mm]	$\pm$ 420
maximum rotation [rad]	0.006
maximum dynamic friction coefficient (dry)	0.12

Table 3: List of sensors in the CWH

location in the building	sensor location	measurement	sensors	direction(s)

NE corner				
	6th floor	acceleration	tri-axial	x, y, z
	lower ground (above BI)	acceleration	uni-axial	y
	foundation (below BI)	acceleration	tri-axial	x, y, z
	base isolators	displacement	string potentiometer	y
SW corner				
	6th floor	acceleration	tri-axial	x, y, z
	lower ground (above BI)	acceleration	two uni-axial	x, y
	foundation (below BI)	acceleration	tri-axial	x, y, z
	base isolators	displacement	two string potentiometers	x, y

Table 4: Design parameters of base isolation system

parameter	**design value**	**definition**
D_y	10 mm	isolator yield displacement
C_y	0.0286	yield strength coefficient
κ	0.1574	post-yield stiffness ratio

Table 5: Summary of equivalent parameters of models

structural parameters	**SDOF (nonlinear)**	**SDOF (linear)**	**2DOF**	**3DOF**
stiffness super structure [N/m]	--	--	$k_s = 8.63 \cdot 10^8$	$k_l = 8.12 \cdot 10^8$ $k_u = 2.95 \cdot 10^8$
stiffness base-isolator [N/m]	PWA parameters (Table 4)	$k_b = 7.38 \cdot 10^8$ (design) $k_b = 2.96 \cdot 10^9$ (adjusted)	$k_b = 2.96 \cdot 10^9$	$k_b = 2.96 \cdot 10^9$
mass distribution of superstructure [kg]	scaled with respect to mass	$m_{tot} = 2.7 \cdot 10^7$	$m_{aboveBI} = 0.16\, m_{tot}$	$m_{aboveBI} = 0.1\, m_{tot}$, $m_l = 0.6\, m_{tot}$, $m_u = 0.3\, m_{tot}$
damping ratios per mode	20%		5% (both modes)	10% (1st mode), 15% (2nd and 3rd mode)

Table 6: Peak acceleration comparison of modeled and measured data for 3DOF case.

seismic event	sensor location	measured peak acceleration [mm/s ^{2}]	modeled peak acceleration [mm/s ^{2}]	difference [%]
M_w6.0				
	above base isolator	1201	1097	8.7
	6th floor	1742	1547	11.0
M_w5.8				
	above base isolator	771	827	7.3
	6th floor	1536	1644	7.0